PROJECT NO.: SM 302079.GIS

January 14, 2022
Revised May 21, 2026

CUADRO ARCH LTD.
12 Madison Avenue
Toronto, Ontario
M5R 2S1

Attention: Elias Ghasemian

GEOTECHNICAL INVESTIGATION
PROPOSED TWO STOREY DATA CENTRE
3110 SOUTH SERVICE ROAD
BURLINGTON, ONTARIO

Dear Mr. Ghasemian,

Further to your authorisation, SOIL-MAT ENGINEERS & CONSULTANTS LTD. has completed the fieldwork, laboratory testing, and report preparation in connection with the above noted project. The scope of work was completed in general accordance with our proposal P302079 dated August 13, 2021. Our comments and recommendations based on our findings at the eight [8] borehole locations are presented in the following paragraphs.

1. INTRODUCTION

We understand that the project will involve the demolition of the existing structure and construction of a new two storey data center structure with one underground parking level at 3110 South Service Road in Burlington, Ontario. The purpose of this geotechnical investigation work is to assess the subsurface soil and groundwater conditions, and to provide our comments and recommendations with respect to the design and construction of the proposed development, from a geotechnical point of view.

It is noted that our office has prepared a Phase One Environmental Site Assessment [ESA] Update for the subject property [SM 302079-E dated March 11, 2026] which has been provided under a separate cover. This report is based on the above summarised project description, and on the assumption that the design and construction will be performed in accordance with applicable codes and standards.

Any significant deviations from the proposed project design may void the recommendations given in this report. If significant changes are made to the proposed design, this office must be consulted to review the new design with respect to the results of this investigation.

2. PROCEDURE

A total of eight [8] sampled boreholes were advanced at the locations illustrated in the attached Drawing No. 1, Borehole Location Plan. The boreholes were advanced using continuous flight power auger equipment on October 27, 2021 under the direction of a staff member of SOIL-MAT ENGINEERS & CONSULTANTS LTD., to depths of between approximately 1.7 and 5.2 metres below the existing ground surface.

Upon completion of drilling, a monitoring well was installed in Borehole No. 3. The monitoring wells consist of 50-millimetre PVC pipe, screened in the lower 1.5 metres. The monitoring well was installed to a depth of approximately 4.7 metres and encased in well filter sand up to approximately 0.3 metres above the screened portion, then with bentonite 'hole plug' to the surface and fitted with a protective steel 'flush mount' casing. The remaining boreholes were backfilled in general accordance with Ontario Regulation 903, and the ground surface was reinstated flush with the surrounding grade.

Representative samples of the subsoils were recovered from the borings at selected depth intervals using split barrel sampling equipment driven in accordance with the requirements of ASTM test specification D1586, Standard Penetration Resistance Testing. After undergoing a general field examination, the soil samples were preserved and transported to the SOIL-MAT laboratory for visual, tactile, and olfactory classifications. Routine moisture content tests were performed on all soil samples recovered from the borings, with hand penetrometer testing conducted on all cohesive samples.

The boreholes were located in the field by a representative of SOIL-MAT ENGINEERS & CONSULTANTS LTD. The ground surface elevation at the borehole locations were provided by the topographic drawing prepared by J.H. Gelbloom Surveying Limited [Project No. 20-265], provided to our office. Details of the conditions encountered in the boreholes, together with the results of the field and laboratory tests, are presented in Log of Borehole Nos. 1 to 8 inclusive, following the text of this report. It is noted that the boundaries of soil types indicated on the borehole logs are inferred from non-continuous soil sampling and observations made during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and therefore should not be construed at the exact depths of geological change.

3. SITE DESCRIPTION AND SUBSURFACE CONDITIONS

The subject site is located at 3110 South Service Road, Burlington, Ontario. The subject site consists of an existing single story slab on grade commercial structure central to the property with surrounding asphaltic concrete paved parking and landscaped areas. The site is bounded to the south and east by existing commercial development, to the west by the South Service Road, and to the north by the South Service Road and Queen Elizabeth Way [QEW] considering an east-west orientation of the QEW. The grade is generally flat and even, with a total elevation variation of less than 1 metre.

The subsurface conditions encountered at the borehole locations are summarised as follows:

Pavement Structure

Borehole Nos. 1 through 8 were advanced through the existing asphalt pavement structure. The pavement structure at the borehole locations consisted of approximately 100 millimetres of asphaltic concrete over 150 millimetres of compact granular material. It is noted that the pavement structure across the site may vary from what is noted at the borehole locations. The asphaltic concrete was noted to be generally in a poor condition with frequent longitudinal, transverse, and alligator cracking, as well as evidence of numerous patches and repairs to the pavement, concentrated along sewer lines and at the entrances to the property.

Clayey Silt Fill

Clayey silt fill was encountered below the pavement structure at Borehole Nos. 2, 3, 4, and 5. The cohesive fill soils were brown to reddish brown in colour, contained trace to some sand and gravel, and were generally firm in consistency. The cohesive soils encountered were proven to depths of between approximately 1.0 to 1.5 metres below the existing ground surface at Borehole Nos. 3, 4, and 5. It is likely that the fill materials are generated from the site as part of site grading associated with the original development of the property. It is noted that the depth of fill materials may vary across the site.

Clayey Silt Till

Native clayey silt till was encountered below the pavement structure and/or fill material at all borehole locations. The native fine grained to cohesive soils were reddish brown in colour, contained trace to some sand, occasional shale fragments, and were generally stiff to very stiff in consistency. The native till soils encountered were proven to depths of approximately 1.6 to 3.6 metres below the existing ground surface, to termination or transition to weathered Queenston shale bedrock.

Queenston Shale Bedrock

Weathered Queenston shale bedrock was encountered beneath the clayey silt till at Boreholes Nos. 1 to 5, at depths of approximately 1.6 to 2.8 metres. The Queenston shale bedrock was severely weathered in the upper levels, becoming more sound with depth, and was hard in terms of soil with occasional harder grey seams. It is noted that the severely weathered upper levels of the Queenston shale exhibits characteristics of a very stiff to hard cohesive soil, making the transition from the clayey silt till overburden soils to weathered Queenston shale bedrock somewhat indistinct. It is noted that the Queenston shale bedrock was not cored during this investigation. The approximate depths and elevations that the Queenston shale bedrock have been summarized in the table below.

TABLE A – WEATHERED QUEENSTON SHALE DEPTHS

Borehole No.	Ground Surface Elevation (m)	Depth of Weathered Queenston Shale Below Ground Surface (m)	Weathered Queenston Shale Elevation (m)
BH 1	105.85	1.6	104.3
BH 2	105.98	2.3	103.7
BH 3	106.02	2.3	103.7
BH 4	106.1	2.8	103.3
BH 5	105.89	2.3	103.6

Grain Size Analyses

Grain size analyses were conducted on two [2] selected samples of the native soils recovered from the boreholes. The results of this grain size testing can be found appended to the end of this report, and are summarized as follows:

TABLE B – GRAIN SIZE ANALYSES

Sample ID	Depth	% Clay	% Silt	% Sand	% Gravel	Hydraulic Conductivity, k [cm/s]	Estimated Infiltration Rate, [mm/hr]
BH1 SS2	1.5 m	18	65	16	0	10^{-7}	<10
BH2 SS1	0.8 m	23	69	8	0	10^{-8}	<10

The results of the lab testing indicate the soils to consist predominately of clay and silt, with trace to some sand. These soils are relatively impermeable, with hydraulic conductivities on the order of 10^{-7} to 10^{-8} cm/sec. According to the Unified Soil Classification System (USCS), the soils are classified as M.L. – clayey silts with slight plasticity. These results of consistent with our observations of the soils during drilling and visual assessment of the recovered samples.

A review of available published information [Quaternary Geology of Ontario, Southern Sheet Map 2556] indicate the subsurface soils to consist of clay to silt-textured till overlying Queenston shale bedrock at relatively shallow depths, consistent with our experience in the area and observations during drilling. It is noted too, that the presence of occasional harder layers within the Shale is anticipated, typically consisting of grey limestone and possibly sandstone. The presence of such harder grey layers, often referred to as limestone stringers, are anticipated to be present in the bedrock. These harder layers vary in thickness, but are typically on the order of 50 to 200 millimetres in thickness.

Groundwater Observations

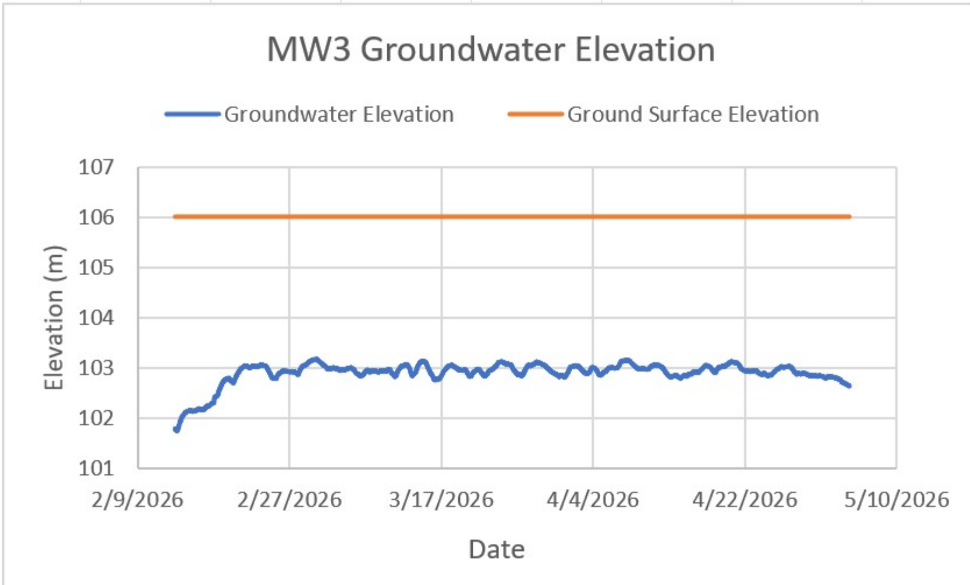
All of the boreholes were recorded as 'dry' upon completion with the exception of Borehole Nos. 2 and 3 where freestanding water was encountered at approximate depths of 4.4 and 4.6 metres respectively. It is noted that insufficient time would have passed for the static groundwater level to stabilise in the open boreholes. As noted above, a monitoring well was installed in Borehole No. 3, to allow for measurements of the static groundwater level.

The water level in the monitoring well was measured with data logger equipment as well as manually measured on December 21, 2021, February 13, 2026 and May 4, 2026 which have been summarized below.

TABLE C – MONITORING WELL READINGS

Date		December 21, 2021		February 13, 2026		May 4, 2026	
Monitoring Well No.	Surface Elevation (m)	Ground water Depth (m)	Ground water Elevation (m)	Ground water Depth (m)	Ground water Elevation (m)	Ground water Depth (m)	Ground water Elevation (m)
3	106.02	3.48	102.54	4.01	102.01	3.38	102.64

Data loggers were also installed at monitoring well No.3 to allow for continuous measurement of the groundwater level between February 13, 2026 to May 4, 2026. The data obtained from these data loggers have been illustrated as follows:



Based on the water levels measured by data logger and manually the groundwater level varies from approximately 3.0 to 4.0 metres, with an average value during the monitoring period of approximately 3.5 metres. For design purposes the seasonal high groundwater level as measured between February and May 2026 as illustrated below, is estimated at a depth of approximately 3 metres below the surrounding grade, at an elevation of 103.0 metres.

4. FOUNDATION CONSIDERATIONS

The subsurface conditions encountered at the borehole locations are considered to be capable of supporting the proposed structures on conventional spread footings founded on the native clayey silt till or Queenston shale below any fill or otherwise unsuitable materials. it is anticipated that footings would be on the order of 1.2 to 1.5 metres below grade, with the native clayey silt. With up to one underground parking level, it is anticipated that the proposed founding depths for the proposed budling, will be on the order of 3.0 to 3.5 metres below the existing grade. This will result in a founding level extending into the weathered to competent Queenston shale bedrock.

Spread footings founded in the native overburden soils or Queenston Shale bedrock may be designed considering the following conservative bearing values:

Native Clayey silt till/Weathered Shale [<2.5 metre below grade]:
SLS = 250 kPa [~5,000 psf]
ULS = 400 kPa [~8,000 psf]

Sound Shale [>2.5 metre below existing grade]
SLS = 750 kPa [~15,000 psf]
ULS = 750 kPa [~15,000 psf]

It is noted that higher bearing pressures would likely be available for the competent Queenston shale bedrock and would have to be confirmed through coring and unconfined compression tests on the selected recovered bedrock cores. However, it is not expected that such higher bearing values would be required for the proposed scale of building. The footing beds should be cleaned of any loose or disturbed material, along with any ponded water, immediately prior to the placement of foundation concrete.

It is noted that the SLS value represents the Serviceability Limit State, which is governed by the tolerable deflection [settlement] based on the proposed building type, using unfactored load combinations. The ULS value represents the Ultimate Limit State and is intended to reflect an upper limit of the available bearing capacity of the founding soils in terms of geotechnical design, using factored load combinations. There is no direct relationship between ULS and SLS; rather they are a function of the soil type and the tolerable deflections for serviceability, respectively. Evidently, the bearing capacity values would be lower for very settlement sensitive structures and larger for more flexible buildings. It is also noted that the SLS and ULS bearing capacities are equivalent within the competent Queenston shale bedrock, as in order for the serviceability limits to be realised, ultimate failure of the bedrock would have to occur.

Consideration should be given to the placement of a thin, perhaps 50 millimetres thick, concrete 'mud' slab. This will serve to protect the founding soils from disturbance and provide a clean working surface for the placement of reinforcing steel. In any event, whether mud slab or footing, concrete should be placed over the prepared founding level as soon as possible, ideally within the same day it is excavated and evaluated by a representative from this office.

In areas where it will be necessary to provide adjacent footings at different founding elevations, the lower footing should be constructed before the higher footing is constructed, if possible, and the higher footing should be set below an imaginary line drawn up from the edge of the lower footing at 10 horizontal to 7 vertical. This practice will aid in limiting stress transfer from the higher footings to lower footings.

All footings exposed to the environment must be provided with a minimum of 1.2 metres of earth cover or equivalent insulation to protect against frost damage. This frost protection would also be required if construction were undertaken during the winter months. It is noted that while the basement level is anticipated to be an unheated parking garage space, such spaces are do not typically experience actual sustained freezing air temperatures. Further, the Queenston shale founding conditions would be considered generally non-frost susceptible. As such, interior footings within the parking garage would not require a full 1.2 metres of cover below the basement floor level. It is recommendation that a minimum cover of 0.6 metres would be adequate for interior footings. The noted exception would be at the location of penetrations, such as air vent shafts and entrances, that allow more direct entry of outside air temperatures. At these locations it is recommended that a layer of 50-millimetre thick rigid foam insulation be provided as supplemental protection of footings within perhaps 4 metres of any openings. All footings and foundations should be designed and constructed in accordance with the current Ontario Building Code, and in accordance with the structural drawings and details.

With foundations designed as outlined above and as required by the Building Code, and with careful attention paid to construction detail, total and differential settlements should be small, and certainly well within normally tolerated limits of 25 and 20 millimetres, respectively, for the type of building and occupancy expected.

It is imperative that a soils engineer be retained from this office to provide geotechnical engineering services during the excavation and foundation construction phases of the project. This is to observe compliance with the design concepts and recommendations of this report and to allow changes to be made in the event that subsurface conditions differ from the conditions identified at the Borehole locations.

5. LATERAL EARTH PRESSURE

The lateral earth pressures on basement walls can be estimated on the basis of backfill [free draining granular material] unit weight, [γ], of 19.5 kN/m^3 [~124 pcf]. The coefficient of lateral earth pressure may be taken as, k_o = 0.5 in fill against rigid walls [at rest condition]. Any additional pressures due to surcharge loading, such as parked vehicles, floor slab loading, etc. must be included in the design. Where foundations are constructed as a ‘waterproofed’ section, they must also be designed to support the lateral hydrostatic pressure of a water level at ground surface.

6. SEISMIC DESIGN CONSIDERATIONS

The structure shall be designed according to Section 4.1.8 of the Ontario Building Code, Ontario Regulation 203/24. Based on the subsurface soil conditions encountered in this investigation, the applicable Site Classification for the seismic design is Site Class B – Rock, based on the average soil characteristics for the site. The applicable seismic data from the 2020 National Building Code 2%-in-50 years for the subject site are as follows:

$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	$S_a(5.0)$	$S_a(10.0)$	PGA	PGV
0.31	0.135	0.0631	0.0276	0.00688	0.00248	0.179	0.0883

It is noted that site specific shear wave velocity testing may be conducted to further refin these values.

7. EXCAVATIONS CONSIDERATIONS

With one underground parking level, excavations for the construction of new foundations are expected to extend to depths of about 3 to 4 metres below the existing grade. Such excavations will extend through the clayey silt fill and native clayey silt till overburden, and into the weathered to sound Queenston Shale bedrock. Excavations through the overburden soils and weathered upper levels of the Queenston shale should be readily advanced with heavy hydraulic excavator equipment. Deeper excavations into the sound Shale, below perhaps 2metres or more below existing grade, will become more difficult and the use of mechanical rock splitter equipment may be required, with a corresponding slower rate of excavation.

Excavations through any fill materials would be expected to remain stable at inclinations of up to 45 degrees to the horizontal while the native clayey silt till and weathered Queenston Shale would be expected to remain stable at inclinations of up to 60 degrees to the horizontal, or steeper. Excavations into competent Queenston shale would be expected to remain stable at near vertical inclinations.

Where wet seams are encountered, during periods of extended precipitation, the excavations through any fill materials and native overburden soils may have a tendency to ‘slough in’ to as flat as 3 horizontal to 1 vertical, or flatter, while the competent Queenston shale bedrock should remain stable at near vertical inclinations irrespective of the groundwater conditions. It is recommended that open cut excavations be appropriately sloped in the overburden soils as noted above, and be covered with heavy construction tarps to protect from surface erosion. The near vertical excavation faces in the Queenston shale bedrock should be thoroughly scaled of loose fragments, and similarly provided with a protective tarp.

Notwithstanding the above, all excavations must comply with the current Occupational Health and Safety Act and Regulations for Construction Projects. It is noted that with respect to this act, the fill soils would typically be considered Type 3 soil, native clayey silt till soils and weathered Queenston shale would be considered Type 2 soil. Excavation slopes steeper than those required in the Safety Act must be supported and a senior geotechnical engineer from this office should monitor the work. Support of any existing underground services and roadways adjacent to the project area must also be considered in assessing the excavation support requirements.

Based on the anticipated depth of construction through the Queenston shale bedrock, it may be possible to advance the excavations as open cuts. However, depending on the proximity of the excavations to adjacent structures, property lines, roadways, etc., it may be necessary to provide supplemental support systems or shoring to ensure adjacent structures are not compromised.

As noted above the static groundwater level was measured at approximately 3.0 to 4.0 metres below grade. Excavations above and even slightly below this depth would be expected to be relatively straightforward, with limited groundwater infiltration through the low permeability Queenston shale bedrock. Regardless, infiltration of water through permeable seams as well as surface runoff into open excavations, should still be anticipated. The rate of infiltration is anticipated to be relatively low, such that it should be possible to adequately control for the short construction period using conventional construction dewatering methods, such as pumping from sumps in the base of the excavation. More infiltration should be anticipated when connections are made to existing services, in the area of former structures, etc. Surface water should be directed away from the excavations.

Based on the estimated groundwater levels and anticipated excavation depths, it is expected that minimal, if any, interaction with groundwater would occur, as the estimated groundwater elevation is conservative in nature, and the anticipated excavations are near to above this elevation. Infiltration through the Queenston shale bedrock will also be very slow, likely requiring limited to no dewatering efforts. As such, it will not be necessary to conduct any significant dewatering of groundwater, or be necessary to file an Environmental Activity and Sector Registry [EASR] or Permit to Take Water [PTTW].

8. FLOOR SLAB AND PERIMETER DRAINAGE

As noted above, the seasonal high static groundwater level is estimated as shallow as 3.0 metres depth, at an elevation of roughly 103.0 metres or deeper. It is recommended that the finished floor level of basement space be designed to be sufficiently above the static groundwater level in order to avoid the need for permanent or sustained pumping of groundwater post construction.

In this regard the finished basement floor slab should be designed at an elevation 103.5 metres, or higher. Footing levels could certainly be at lower elevations without requiring long-term dewatering. A water stop should be provided between the footings and foundation walls.

At a minimum, a moisture barrier will be required under the floor slabs such as the placement of at least 200 millimetres of well-compacted 20-millimetre clear crushed stone. At a minimum the moisture barrier material should contain no more than 10 per cent passing the No. 4 sieve. Where 'non-damp' floor slabs are required, as for instance under sheet vinyl floor coverings, etc., extra efforts will be required to damp proof the floor slab, as with the additional provisions of a heavy 'poly' sheet, damp proofing sprays/membranes, drainage board products, etc. Where 'poly' sheets are used care should be taken to prevent puncturing and tearing and a sufficiently heavy gauge material be provided.

In the event that the basement slab elevation is constructed near the seasonal high groundwater level, the associated risk with moisture may be mitigated via the provision of an interior weeping tile system and polyethylene vapour barrier beneath the concrete slab. This would prevent the buildup of water beneath the slab, and help maintain a relatively 'dry' condition during seasonal high groundwater levels. The interior weeping tile should be independent from the exterior weeping tile, and discharge to a separate, oversized sump pit. The provision of a geofabric separator on the exposed subgrade would also serve to prevent 'pumping' of fines up into the void spaces of the clear stone and resultant settlement of the concrete slab. Regardless, a relatively increased frequency of sump pump operations may be experienced during significantly 'wet' seasons.

The basement floor slabs may be constructed using conventional slab-on-grade techniques on a prepared subgrade. The exposed subgrade surface should be well compacted in the presence of a representative of SOIL-MAT ENGINEERS. Any soft 'spots' delineated during this work must be sub-excavated and replaced with quality backfill material compacted to a minimum of 98 per cent of its standard Proctor maximum dry density [SPMDD]. Granular fill, such as an imported Ontario Provincial Standard Specification [OPSS] Granular 'B', Type II (crushed limestone bedrock) product, or approved alternative, is preferred within the building footprint due to its relative insensitivity to weather conditions, ease in achieving the required degree of compaction, and its quick response to applied stresses.

As with all concrete floor slabs, there is a tendency for the floor slabs to crack. The slab thickness, concrete mix design, the amount of steel and/or fibre reinforcement and/or wire mesh placed into the concrete slab, if any, will therefore be a function of the owner's tolerance for cracks in, and movements of, the slabs-on-grade, etc. The 'saw-cuts' in the concrete floors, for crack control, should extend to a minimum depth of 1/3 of the thickness of the slab.

Curing of the slab-on-grade must be carefully specified to ensure that slab curl is minimised. This is especially critical during the hot summer months of the year when the surface of the slab tends to dry out quickly while high moisture conditions in the moisture barrier or water trapped on top of any 'poly' sheet at the saw cut joints and cracks, and at the edges of the slabs, maintains the underside of the slab in a moist condition.

It is important that the concrete mix design provide a limiting water/cement ratio and total cement content, which will mitigate moisture related problems with low permeance floor coverings, such as debonding of vinyl and ceramic tile. It is equally important that excess free water not be added to the concrete during its placement as this could increase the potential for shrinkage cracking and curling of the slab.

The basement foundation walls should be suitably damp proofed, including the provision of a 'dimple type' drainage board to promote rapid drainage to a perimeter drainage system. The perimeter drainage system should consist of 100-millimetre diameter perforated pipe, encased in a geofabric sock and covered with a minimum of 200 millimetres of a 20-millimetre clear crushed stone product, and the clear crushed stone in turn encased by a heavy filter geotextile product. The suppliers of the filter geotextile should be consulted as to the type best suited for this project. Care should be taken during the installation of the drains as even a small break in the filtering materials could result in loss of fines into the drains with attendant performance difficulties, including settlements of the ground surface. The perimeter drains should outlet to a gravity sewer connection, a nearby catch basin, or a sump pit a minimum of 150 millimetres below the underside of finished floor. The exterior grade around the structure should be sloped away from the structure to prevent the ponding of water against the foundation walls. The enclosed Drawing No. 2 shows schematics of the typical requirements for slab-on-grade construction with a basement level and underslab drainage.

Elevator pit excavations extending below the general basement floor level should also be designed to be water-tight. It is recommended that this water-tight design for the elevator pit be implemented regardless of the water proofing or drainage design adopted for the general basement level.

9. BACKFILL CONSIDERATIONS

The excavated material will consist primarily of the clayey silt fill, native clayey silt till, and Queenston shale bedrock, encountered in the boreholes, as described above. The clayey silt fill and native soils are generally considered suitable for use as engineered fill, trench backfill, etc. in settlement sensitive area, provided that they are free of organics or other deleterious material, and that its moisture content can be controlled to within 3 per cent of its standard Proctor optimum moisture content.

The upper levels of the highly weathered Queenston shale may be feasible for use backfilling settlement sensitive areas, depending on the degree of weathering. However, significant effort to properly break down and moisture condition will be critical to achieving an effective compaction operation.

While it is possible to use the excavated competent Queenston shale as backfill in services trenches, etc. the use of shale fill material is cautioned as it can be prone to long-term ongoing settlements. Significant effort is required by the contractor to sufficiently break down the excavated bedrock into a well graded material and large volumes of water are required in order to properly moisture condition the material and achieve an effective compaction operation. In general, the use of shale fill is not recommended in settlement sensitive areas.

It is noted that the clayey silt soils and weathered Queenston shale bedrock are not considered to be free draining and should not be used where this characteristic is necessary. It is also noted that these fine grained to cohesive soils will present difficulties in achieving effective compaction where access with compaction equipment is restricted, such as within the building footprint or against foundations. The on-site soils encountered are generally considered to be near to ‘dry’ of their standard Proctor optimum moisture content. Some moisture conditioning may be required depending upon the weather conditions at the time of construction, especially where the material contains inclusions of shale. The fine grained to cohesive soils are also noted to be sensitive to high moisture conditions and will become almost impossible to effectively compact when they become well ‘wet’ of optimum. After a period of heavy precipitation, any near-surface wet or softened material should be allowed to air dry or be removed and discarded.

The use of free draining, well-graded granular material, such as an Ontario Provincial Standard Specification [OPSS] Granular B, Type II (crushed bedrock), or approved alternative, is recommended for backfill against foundation walls or to raise the interior grade to the design subgrade level. This material is more readily compacted in restricted access areas, and generally presents a more positive support condition for interior floor slabs and exterior concrete sidewalks.

We note that where backfill material is placed near or slightly above its optimum moisture content, the potential for long term settlements due to the ingress of groundwater and collapse of the fill structure is reduced. Correspondingly, the shear strength of the 'wet' backfill material is also lowered, thereby reducing its ability to support construction traffic and therefore impacting roadway construction.

If the soil is well dry of its optimum value, it will appear to be very strong when compacted, but will tend to settle with time as the moisture content in the fill increases to equilibrium condition. The clayey silt soils may require high compaction energy to achieve acceptable densities if the moisture content is not close to its standard Proctor optimum value.

It is therefore very important that the placement moisture content of the backfill soils be within 3 per cent of its standard Proctor optimum moisture content during placement and compaction to minimise long term subsidence [settlement] of the fill mass. Any imported fill required in service trenches or to raise the subgrade elevation should have its moisture content within 3 per cent of its optimum moisture content and meet the necessary environmental guidelines.

A representative of SOIL-MAT should be present on-site during the backfilling and compaction operations to confirm the uniform compaction of the backfill material to project specification requirements. Close supervision is prudent in areas that are not readily accessible to compaction equipment, for instance near the end of compaction 'runs'. All structural fill, areas to be paved, or otherwise settlement sensitive areas should be compacted to a minimum of 98 per cent of its standard Proctor maximum dry density [SPMDD]. The appropriate compaction equipment should be employed based on soil type, i.e. pad-toe for cohesive soils and smooth drum/vibratory plate for granular soils. A method should be developed to assess compaction efficiency employing the on-site compaction equipment and backfill materials during construction.

10. PAVEMENT STRUCTURE DESIGN CONSIDERATIONS

It is anticipated that portions of the pavement structure will be reconstructed as part of the project, specially the northern portion outside of the proposed building footprint. The existing pavement structure was noted to be generally in a poor condition, exhibiting regular longitudinal, transverse, and alligator cracking, as well as evidence of numerous patches and repairs to the pavement, concentrated along sewer lines and at the entrances to the property. This poor pavement condition is consistent with a relatively limited depth of granular base, and cohesive subgrade soils, present in the boreholes.

Given the thickness and condition of the asphalt and granular base materials, a full depth reconstruction would be considered appropriate. Full depth reconstruction would provide the longest lifespan and lower maintenance requirement, yielding a lower life-cycle cost despite a higher initial cost versus partial depth reconstruction.

FULL DEPTH RECONSTRUCTION

A full depth reconstruction would involve the removal of the existing asphaltic concrete layers and granular base materials to the proposed subgrade elevation. This approach would provide for a full depth of granular base material, with a resulting increased lifespan, and would maintain the existing grades of the roadway. The existing asphaltic concrete could be pulverised into the existing granular base, and possibly mixed in with new imported granular fill material, to create a well-graded granular product that could be re-used on the project as trench backfill, subgrade stabilization material or sub-base course material, depending on the material gradation.

Laboratory sieve analyses would be required to assess the suitability of the pulverised/existing granular materials. The subgrade level would then be lowered, where required, to accommodate the depths of the new pavement structure and finished grade.

The exposed subgrade should be well compacted and proofrolled with 3 to 4 passes of a loaded tandem-axle dump truck [over the same wheel path] in the presence of a representative of SOIL-MAT immediately prior to the placement of the sub-base material. Any areas of distress revealed by this or other means should be sub-excavated and replaced with suitable backfill material, such as with suitable on-site granular materials or imported OPSS Granular ‘B’, Type II, compacted to a minimum of 98 per cent of its SPMDD. Alternatively, the soft areas may be repaired by ‘punching’ coarse aggregate, such as a 50-millimetre clear crushed stone, or reclaimed asphalt and granular materials into the soft areas. The need for sub-excavation of softened subgrade materials will be reduced if construction is undertaken during the dry summer months of the year and careful attention is paid to the compaction operations.

Good drainage provisions will optimise the long-term performance of the pavement structure. The subgrade must be properly crowned and shaped to promote drainage to the subdrain system. Subdrains should be installed to intercept excess subsurface water and to prevent softening of the subgrade material. Surface water should not be allowed to pond adjacent to the outer limits of the paved areas. The most severe loading conditions on the subgrade typically occur during the course of construction, therefore precautionary measures may have to be taken to ensure that the subgrade is not unduly disturbed by construction traffic.

The suggested pavement structures outlined in Table D are based on subgrade parameters estimated on the basis of visual and tactile examinations of the on-site soils and past experience. The outlined pavement structure may be expected to have an approximate ten-year life, assuming that regular maintenance is performed. Should a more detailed pavement structure design be required, site specific traffic information would be needed, together with detailed laboratory testing of the subgrade soils.

TABLE D - RECOMMENDED PAVEMENT STRUCTURES

LAYER DESCRIPTION	COMPACTION REQUIREMENTS	LIGHT DUTY SECTIONS	HEAVY DUTY [TRUCK ROUTE]
Asphaltic Concrete Wearing course OPSS HL 3 or HL 3A	92 per cent Marshall MRD	40 millimetres	40 millimetres
Binder Course OPSS HL 8	92 per cen+t Marshall MRD	50 millimetres	80 millimetres
Base Course OPSS Granular A	100% SPMDD	150 millimetres	150 millimetres
Sub-base Course OPSS Granular B Type II	100% SPMDD	300 millimetres	450 millimetres

* Marshall MRD denotes Maximum Relative Density.

* SPMDD denotes Standard Proctor Maximum Dry Density, ASTM-D698.

Depending on the anticipated traffic, a reduced light duty asphalt structure consisting of 65 millimetres of HL3 surface course may also perform sufficiently. This would be reasonable in areas subjected only to light vehicles such as cars for parking. Such a structure may have a reduced lifespan if subjected to heavier vehicles, and would also not allow for ‘mill and pave’ type operations for future rehabilitation.

Where asphalt pavement is to be constructed above the roof deck of the below grade parking level, the granular base layers recommended for the light duty pavement structure recommended above may be considered for both light duty and heavy duty areas. It is noted that in such cases the roof deck slab should be sufficiently sloped and/or provided with suitable subdrains, in order to promote rapid drainage of water from beneath the pavement. As well the roof slab should be provided with a suitable water proofing system.

It may be feasible to reuse the granular material from the existing pavement structure within the new pavement structure, provided it can be sufficiently separated from the underlying subgrade soils. Depending on the application of the reused granular material, specific laboratory sieve analysis may be warranted. This would be best assessed at the time of construction.

To minimise segregation of the finished asphalt mat, the asphalt temperature must be maintained uniform throughout the mat during placement and compaction. All too often, significant temperature gradients exist in the delivered and placed asphalt with the cooler portions of the mat resisting compaction and presenting a honeycomb surface. As the spreader moves forward, a responsible member of the paving crew should monitor the pavement surface, to ensure a smooth uniform surface. The contractor can mitigate the surface segregation by 'back-casting' or scattering shovels of the full mix material over the segregated areas and raking out the coarse particles during compaction operations. Of course, the above assumes that the asphalt mix is sufficiently hot to allow the 'back-casting' to be performed.

11. SOIL EXPORT CONSIDERATIONS

It is expected that the construction of an underground level will result in the generation of considerable volumes of excess soil requiring off-site disposal, which is governed by Ontario Regulation 406/19. The Regulation requires site specific environmental assessment and stringent testing based on volume to support off-site disposal. SOIL-MAT ENGINEERS may be retained to conduct the necessary assessment, testing, and reporting in accordance with the Regulation once detailed pertaining to off-site disposal are known.

12. LOW IMPACT DEVELOPMENT [LID] CONSIDERATIONS

The soil conditions encountered consisted of a thin layer of clayey silt overburden soils with Queenston shale bedrock at relatively shallow depths. The overburden cohesive soils and Queenston shale bedrock are effectively impermeable, and as such not capable of supporting low impact development [LID] stormwater management systems such as infiltration galleries, etc., which would rely on infiltration within the native soils.

13. GENERAL COMMENTS

The comments provided in this document are intended only for the guidance of the design team. The material in it reflects SOIL-MAT ENGINEERS' best judgement in light of the information available to it at the time of preparation. The subsurface descriptions and test pit information are intended to describe conditions at the borehole locations only. It is the contractors' responsibility to determine how these conditions will affect the scheduling and methods of construction for the project. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. SOIL-MAT ENGINEERS accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We trust that this geotechnical report is sufficient for your present requirements. Should you require any additional information or clarification as to the contents of this document, please do not hesitate to contact the undersigned.

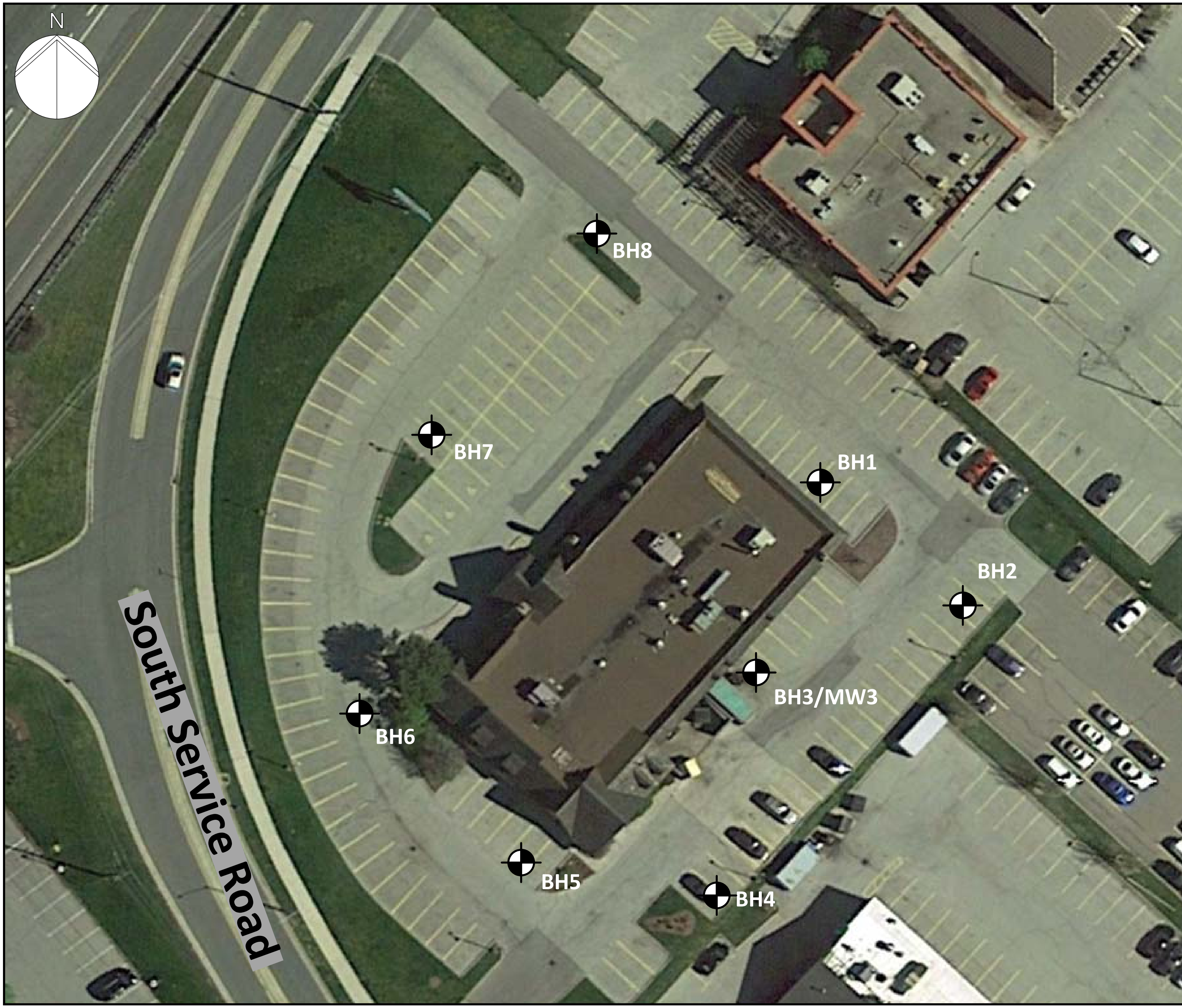
Yours very truly,
SOIL-MAT ENGINEERS & CONSULTANTS LTD.

Yash Parekh, B.Eng., M.Eng.
Junior Geotechnical Associate

Kyle Richardson, P.Eng., QP_{ESA}
Senior Engineer

Enclosures: Drawing No. 1, Borehole Location Plan
Log of Borehole Log Nos. 1 to 8, inclusive
Grain Size Analyses
Drawing No. 2, Typical Design Requirements, Basement Foundations with Under Floor Drains

Distribution: Cuadro Arch Ltd. [pdf]



LEGEND

Borehole Location
BH#

NOTES

1. This drawing should be read in conjunction with Soil-Mat Engineers & Consultants Ltd. Report No. SM 302079-G.

2. Borehole locations are approximate.

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ENGINEERS & CONSULTANTS LTD.

Proposed Addition
3110 South Service Road
Burlington, Ontario

Borehole Location Plan

Project No. SM 302079-G

Date: November 2021

Drawn: MC | Checked: IS

SM 302079-G Borehole Location Plan

Drawing No. 1

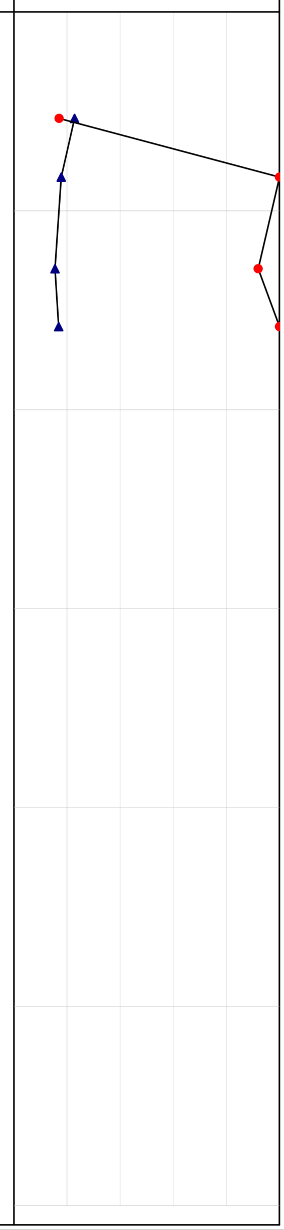
Log of Borehole No. 01

Project No: SM 302079-G
Project: Proposed Commercial Building Addition
Location: 3110 South Service Road, Burlington
Client: Ruth Victor & Associates

Project Manager: Ian Shaw, P.Eng.
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4800885
E: 597279

Depth (m)	Elevation (m)	Description	Well Data	Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0	105.85	Ground Surface								
	105.60	**Pavement Structure** Approximately 100 millimetres of asphaltic concrete over 150 millimetres of compact granular base.								
				SS	1	5,7,10,18	17			
	104.30	**Clayey Silt Till** Reddish Brown, trace to some sand, occasional shale fragments, reworked in the upper levels, very stiff.		SS	2	29,50/5"	100			
		Queenston Shale Red, with occasional harder grey layers, highly weathered in upper levels, becoming more sound with depth, hard.		SS	3	12,44 48,50/5"	92			
	102.60			SS	4	18,50/3"	100			
		End of Borehole								

Moisture Content w% (10, 20, 30, 40)
Standard Penetration Test blows/300mm (20, 40, 60, 80)



NOTES:

1. Borehole was advanced using solid stem auger equipment on October 27, 2021 to spoon refusal on Queenston shale bedrock at a depth of 3.3 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Augers
Drill Date: October 27, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Elements Geo

Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
E: info@soil-mat.ca

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Checked by: IS
Sheet: 1 of 1

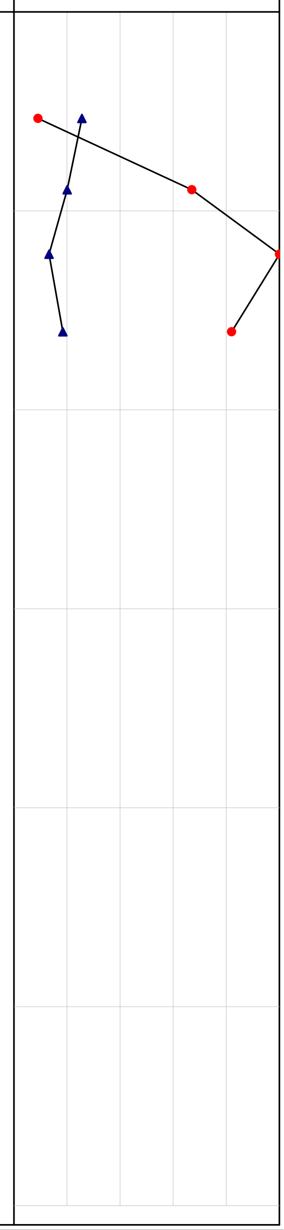
Log of Borehole No. 02

Project No: SM 302079-G
Project: Proposed Commercial Building Addition
Location: 3110 South Service Road, Burlington
Client: Ruth Victor & Associates

Project Manager: Ian Shaw, P.Eng.
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4800845
E: 597309

Depth (m)	Elevation (m)	Description	Well Data	Sample Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0	105.98	Ground Surface								
	105.73	**Pavement Structure** Approximately 100 millimetres of asphaltic concrete over 150 millimetres of compact granular base.								
	104.98	**Clayey Silt Fill** Reddish brown, some sand and gravel, stiff.		SS	1	3,4,5,16	9			
				SS	2	14,21 46,50/3"	67			
	103.70	**Clayey Silt Till** Reddish Brown, trace to some sand, occasional shale fragments, stiff to hard.		SS	3	27,50/5.5"	100			
		Queenston Shale Red, with occasional harder grey layers, highly weathered in upper levels, becoming more sound with depth, hard.		SS	4	14,32,50/1"	82			
	101.30	End of Borehole								

Moisture Content w% (10, 20, 30, 40)
Standard Penetration Test blows/300mm (20, 40, 60, 80)



NOTES:

1. Borehole was advanced using solid stem auger equipment on October 27, 2021 to termination at a depth of 4.7 metres.

2. Borehole was recorded as open and 'wet' at a depth of 4.4 metres upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Augers
Drill Date: October 27, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Elements Geo

Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
E: info@soil-mat.ca

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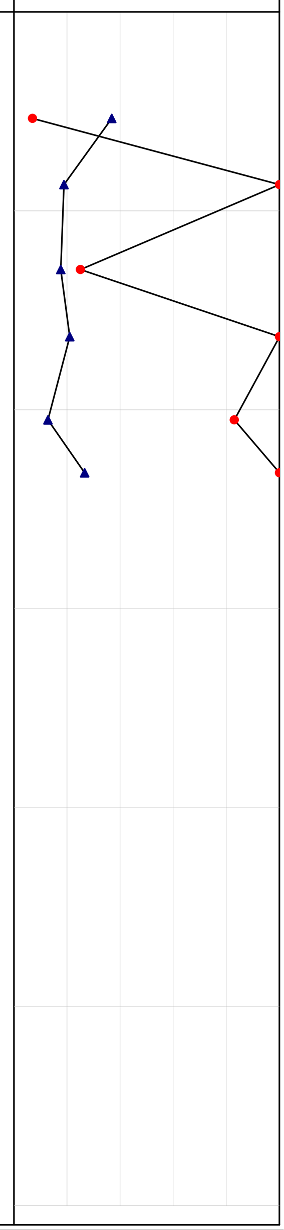
Log of Borehole No. 03

Project No: SM 302079-G
Project: Proposed Commercial Building Addition
Location: 3110 South Service Road, Burlington
Client: Ruth Victor & Associates
Project Manager: Ian Shaw, P.Eng.
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4800833
E: 597289

Depth (m)	Elevation (m)	Description	Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0	106.02	Ground Surface							
	105.77	**Pavement Structure** Approximately 100 millimetres of asphaltic concrete over 150 millimetres of compact granular base.							
			SS	1	3,3,4,7	7			
	104.50	**Clayey Silt Fill** Reddish Brown, trace sand and gravel, firm.	SS	2	30,47,50/5"	100			
	103.70	**Clayey Silt Till** Reddish Brown, trace to some sand, occasional shale fragments, reworked in the upper levels, stiff to hard.	SS	3	15,8,17,41	25			
		Queenston Shale Red, with occasional harder grey layers, highly weathered in upper levels, becoming more sound with depth, hard.	SS	4	12,36,50/5"	100			
			SS	5	40,33 50,50/5"	83			
	101.30		SS	6	50/5"	100			
		End of Borehole							

Moisture Content w% (10 20 30 40)
Standard Penetration Test blows/300mm (20 40 60 80)



NOTES:

1. Borehole was advanced using solid stem auger equipment on October 27, 2021 to termination at a depth of 4.7 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

4. A monitoring well was installed. The following free groundwater level readings have been measured:

December 21, 2021 - 3.48 metres

Drill Method: Solid Stem Augers
Drill Date: October 27, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Elements Geo

Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
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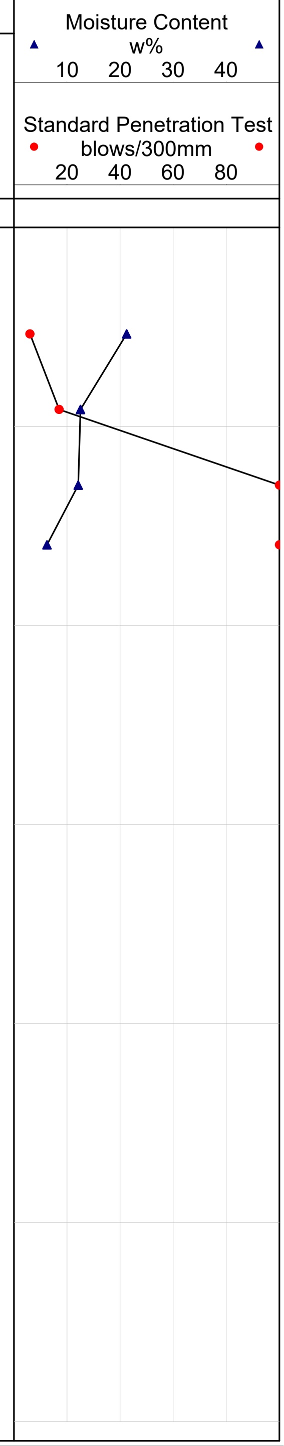
Log of Borehole No. 04

Project No: SM 302079-G
Project: Proposed Commercial Building Addition
Location: 3110 South Service Road, Burlington
Client: Ruth Victor & Associates

Project Manager: Ian Shaw, P.Eng.
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4800805
E: 597278

Depth (m)	Elevation (m)	Description	Well Data	Sample Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0	106.10	Ground Surface								
	105.85	**Pavement Structure** Approximately 100 millimetres of asphaltic concrete over 150 millimetres of compact granular base.		SS	1	4,3,3,7	6			
	104.70	**Clayey Silt Fill** Reddish brown, trace sand and gravel, firm.		SS	2	8,8,9,21	17			
	103.30	**Clayey Silt Till** Reddish Brown, trace to some sand, occasional shale fragments, very stiff to hard.		SS	3	24,36,50/4"	100			
	102.90	**Queenston Shale** Red, with occasional harder grey layers, highly weathered in upper levels, becoming more sound with depth, hard.		SS	4	50,50/5"	100			
		End of Borehole								



NOTES:

1. Borehole was advanced using solid stem auger equipment on October 27, 2021 to termination at a depth of 3.2 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Augers
Drill Date: October 27, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Elements Geo

Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
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Log of Borehole No. 05

Project No: SM 302079-G
Project: Proposed Commercial Building Addition
Location: 3110 South Service Road, Burlington
Client: Ruth Victor & Associates

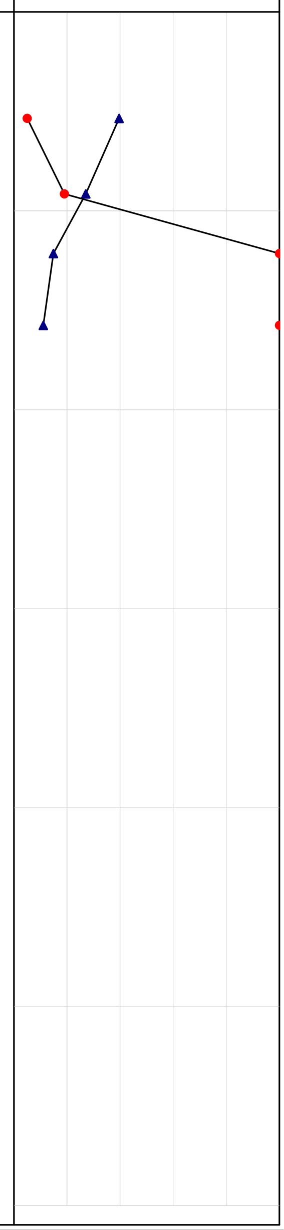
Project Manager: Ian Shaw, P.Eng.
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4800803
E: 597261

Depth (ft / m)	Elevation (m)	Description
0 / 0	105.89	Ground Surface
0–1 ft	105.64	**Pavement Structure** Approximately 100 millimetres of asphaltic concrete over 150 millimetres of compact granular base.
1–5 ft	104.39	**Clayey Silt Fill** Brown, trace sand and gravel, occasional organic staining, firm.
5–8 ft	103.60	**Clayey Silt Till** Reddish Brown, trace to some sand, occasional shale fragments, very stiff to hard.
8–10.5 ft	102.70	**Queenston Shale** Red, with occasional harder grey layers, highly weathered in upper levels, becoming more sound with depth, hard.
		End of Borehole

SAMPLE Type	Number	Blow Counts	Blows/300mm
SS	1	3,2,3,4	5
SS	2	8,9,10,15	19
SS	3	20,50/5"	100
SS	4	50/4"	100

Well Data | Recovery | PP (kgf/cm2) | U.Wt.(kN/m3)

Moisture Content w% 10 20 30 40
Standard Penetration Test blows/300mm 20 40 60 80



NOTES:

1. Borehole was advanced using solid stem auger equipment on October 27, 2021 to termination at a depth of 3.2 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Augers
Drill Date: October 27, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Elements Geo

Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
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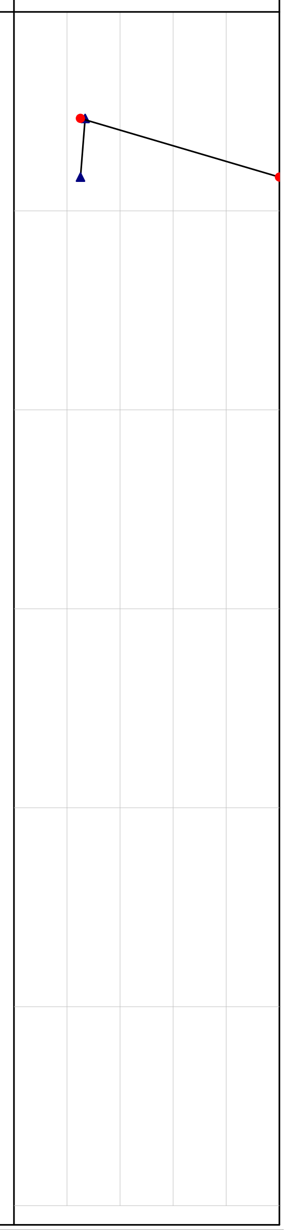
Log of Borehole No. 06

Project No: SM 302079-G
Project: Proposed Commercial Building Addition
Location: 3110 South Service Road, Burlington
Client: Ruth Victor & Associates
Project Manager: Ian Shaw, P.Eng.
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4800825
E: 597237

Depth (m)	Elevation (m)	Description	Well Data	Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0	106.06	Ground Surface								
	105.81	**Pavement Structure** Approximately 100 millimetres of asphaltic concrete over 150 millimetres of compact granular base.								
		Clayey Silt Till Reddish Brown, trace to some sand, occasional shale fragments, stiff to hard.		SS	1	7,10,15,15	25			
	104.20			SS	2	16,50/5"	100			
		End of Borehole								

Moisture Content w% (10, 20, 30, 40) — Standard Penetration Test blows/300mm (20, 40, 60, 80)



NOTES:

1. Borehole was advanced using solid stem auger equipment on October 27, 2021 to termination at a depth of 1.9 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Augers
Drill Date: October 27, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Elements Geo

Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
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Sheet: 1 of 1

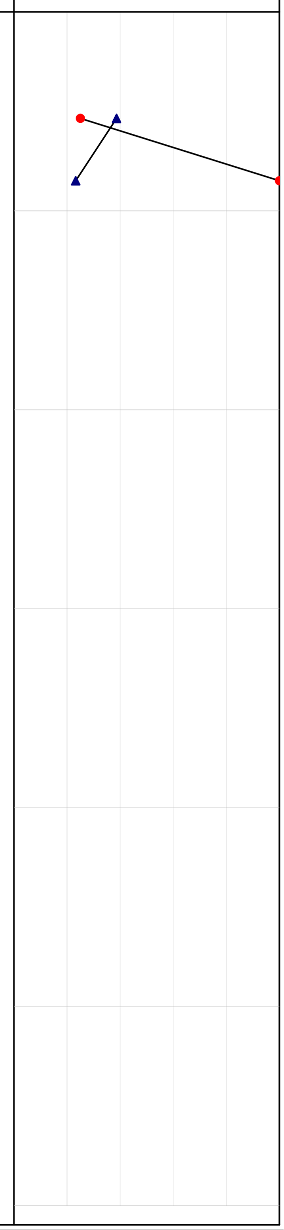
Log of Borehole No. 07

Project No: SM 302079-G
Project: Proposed Commercial Building Addition
Location: 3110 South Service Road, Burlington
Client: Ruth Victor & Associates

Project Manager: Ian Shaw, P.Eng.
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4800858
E: 597245

Depth (ft)	Depth (m)	Elevation (m)	Symbol	Description	Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
0	0	106.11		Ground Surface								
		105.86		**Pavement Structure** Approximately 100 millimetres of asphaltic concrete over 150 millimetres of compact granular base.								
				Clayey Silt Till Reddish Brown, trace to some sand, occasional shale fragments, stiff to hard.		SS	1	4,4,7,8	11			
		104.20				SS	2	12,33,50/2"	100			
				End of Borehole								

Moisture Content w% (10, 20, 30, 40)
Standard Penetration Test blows/300mm (20, 40, 60, 80)



NOTES:

1. Borehole was advanced using solid stem auger equipment on October 27, 2021 to termination at a depth of 1.9 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Augers
Drill Date: October 27, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Elements Geo

Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
E: info@soil-mat.ca

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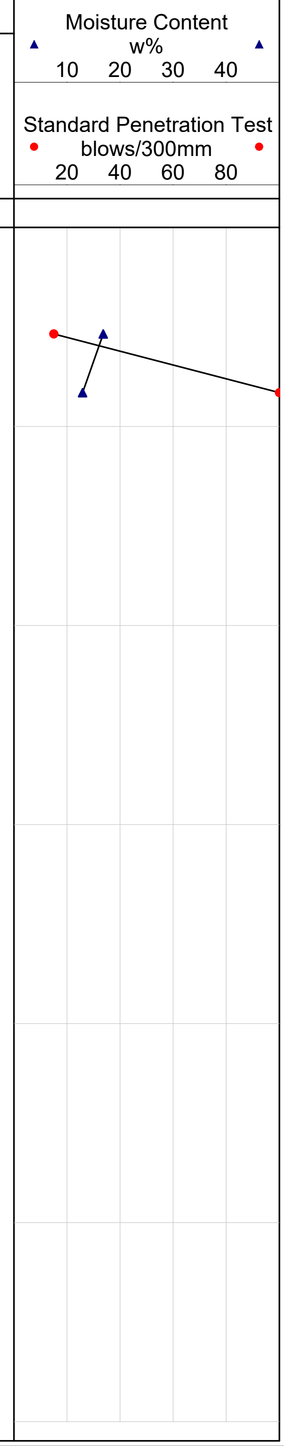
Log of Borehole No. 08

Project No: SM 302079-G
Project: Proposed Commercial Building Addition
Location: 3110 South Service Road, Burlington
Client: Ruth Victor & Associates

Project Manager: Ian Shaw, P.Eng.
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4800884
E: 597265

Depth (m)	Elevation (m)	Description
0	106.13	Ground Surface
0.25	105.88	**Pavement Structure** Approximately 100 millimetres of asphaltic concrete over 150 millimetres of compact granular base.
1.73	104.40	**Clayey Silt Till** Reddish Brown, trace to some sand, occasional shale fragments, very stiff to hard.
		End of Borehole

Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
SS	1	3,6,9,14	15			
SS	2	16,50/4"	100			



NOTES:

1. Borehole was advanced using solid stem auger equipment on October 27, 2021 to spoon refusal on assumed bedrock at a depth of 1.7 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Augers
Drill Date: October 27, 2021
Hole Size: 150 Millimetres
Drilling Contractor: Elements Geo

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130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
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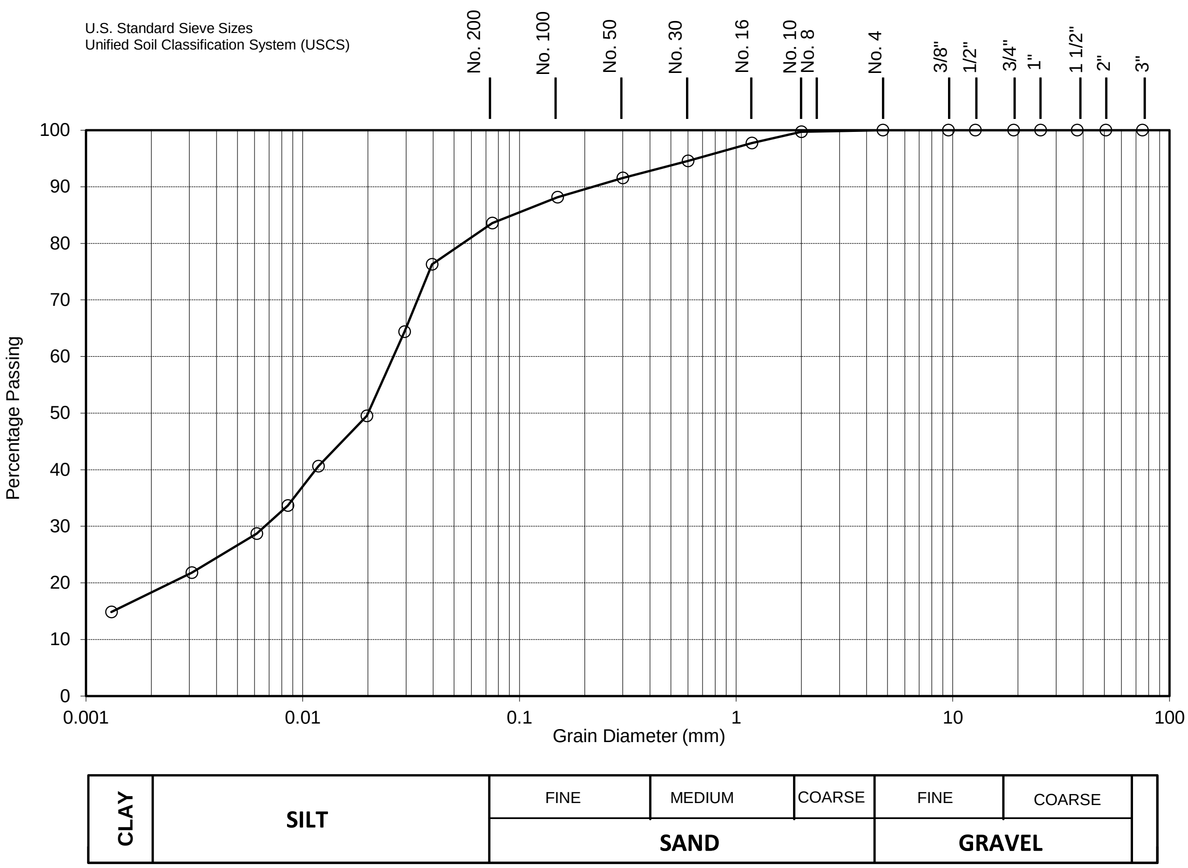
Mechanical & Hydrometer Analyses



Lab No.:	21- 418	Notes: Depth 1.5 m			
Borehole No.:	1				
Sample No:	2				
CLAY [%]:	18	Soil Description:	Reddish Brown Silt w/ some Clay and Sand M.L. - clayey silts with slight plasticity		
SILT [%]:	66				
SAND [%]:	16				
GRAVEL [%]:	0	Estimated Infiltration Rate [mm/hr] :	< 10	Estimated Permeability, k [cm/s]	10^{-7}
D_{10} (Effective Diam. in mm):	0.0008	Coefficient of Uniformity C_U:	32.5	Coefficient of Curvature C_C:	2.2

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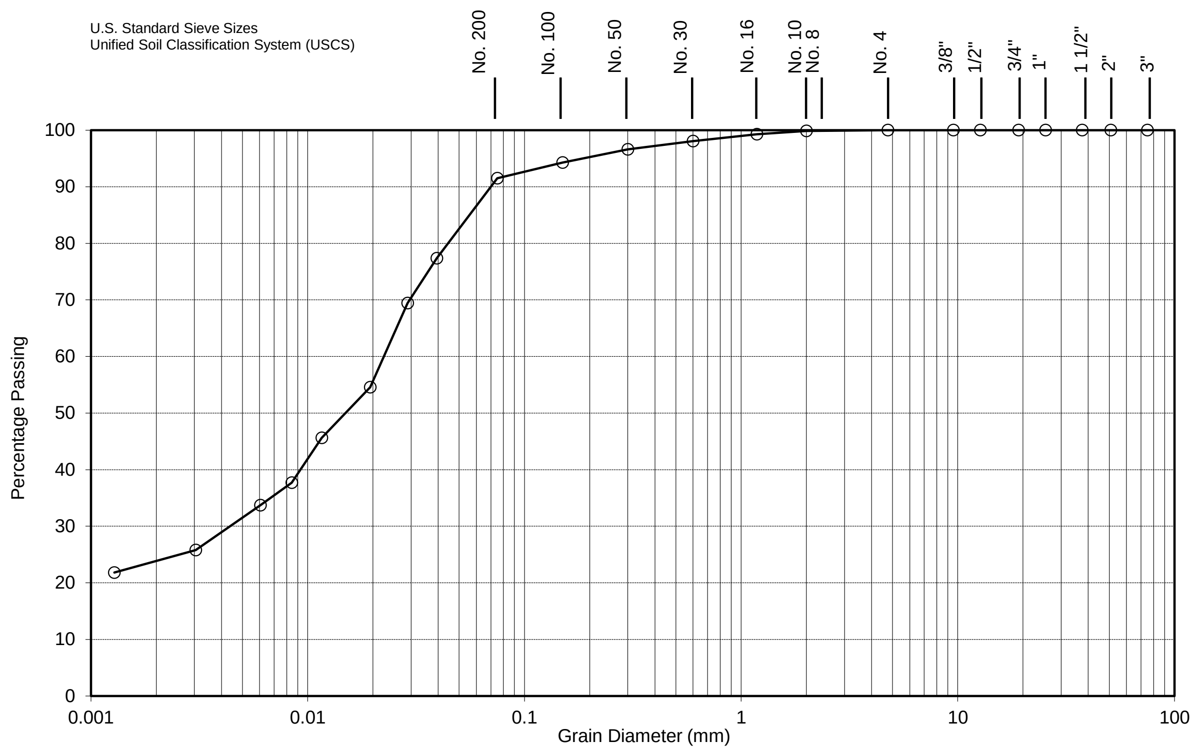
3110 South Service Road, Burlington ON

November 2021	Grain Size Analysis No. 1	Project No.: SM 302079-T

Mechanical & Hydrometer Analyses



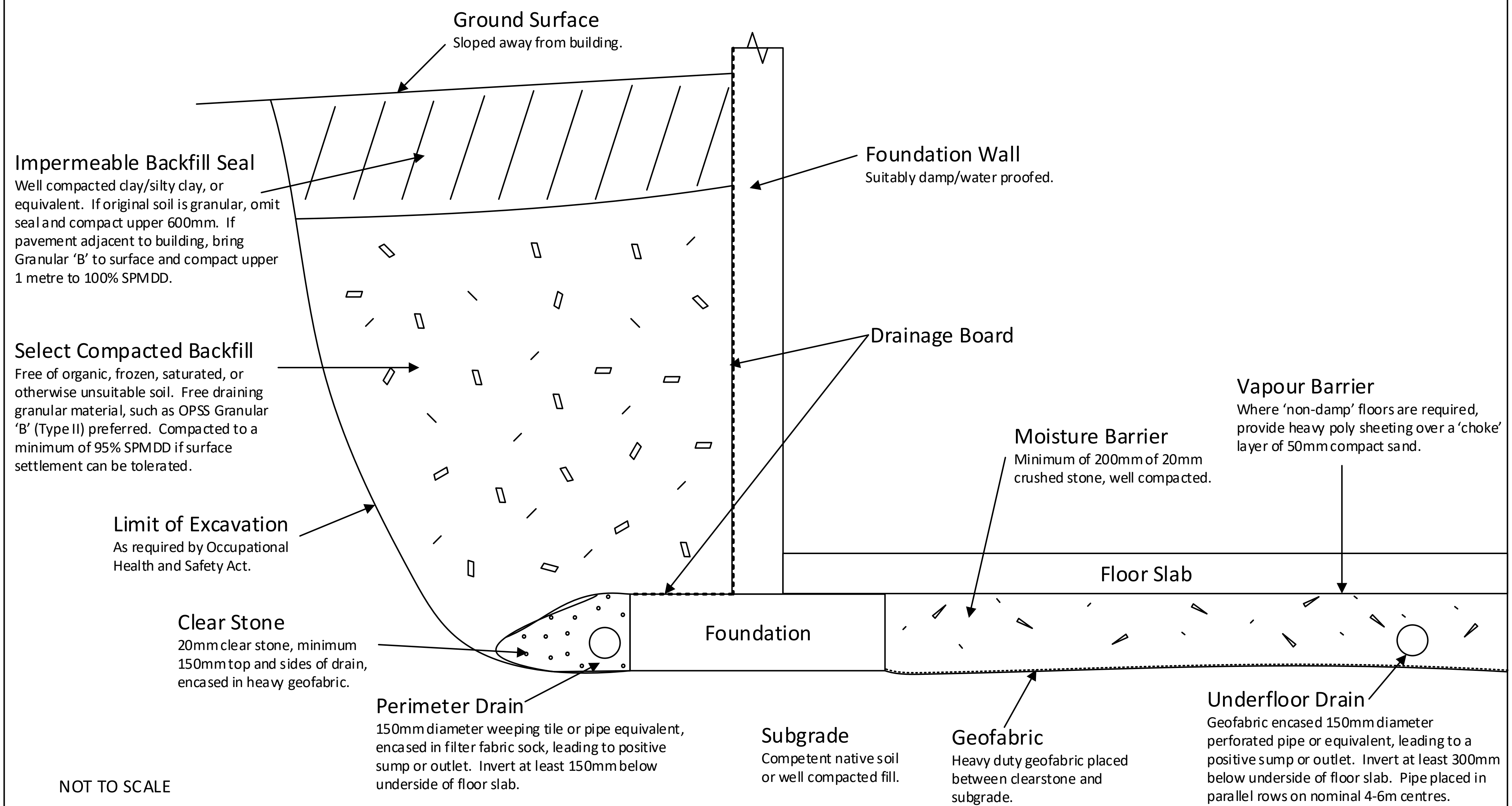
CLAY	SILT	SAND			GRAVEL	
		FINE	MEDIUM	COARSE	FINE	COARSE

Lab No.:	**21- 419**	Notes: Depth 0.8 m			
Borehole No.:	**2**				
Sample No:	**1**				
CLAY [%]:	**23**	Soil Description:	**Reddish Brown Clayey Silt w/ a trace of Sand**		
SILT [%]:	**69**		**M.L. - Clayey silts with slight plasticity**		
SAND [%]:	**8**				
GRAVEL [%]:	**0**	Estimated Infiltration Rate [mm/hr] :	**< 10**	Estimated Permeability, k [cm/s]	10^{-8}
D_{10} (Effective Diam. in mm):	**0.0002**	Coefficient of Uniformity C_U:	**115.0**	Coefficient of Curvature C_C:	**4.4**

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

3110 South Service Road, Burlington ON

November 2021	**Grain Size Analysis No. 2**	**Project No.: SM 302079-T**



Soil-Mat

Soil-Mat Engineers & Consultants Ltd.

Project No.:	SM 302079.GIS
Date:	May, 2026

Typical Design Requirements
Basement Perimeter Drainage with Underfloor Drains – Open Excavation

Drawing No. 2